Effects of alkali-silica reaction and delayed ettringite formation on anchorage of prestressing strands in trapezoidal box beams with dapped ends

Nancy A. Larson, Oguzhan Bayrak, and James O. Jirsa

- Beams originally intended for a highway bridge project were stored in a precasting yard in Houston, Tex., and were exposed to the environment for 15 years. During that period, the concrete experienced deterioration due to alkali-silica reaction and delayed ettringite formation.
- Load tests were conducted on five specimens with varying degrees of distress to study the performance of the dapped end and to assess the effect of the deterioration on the in-service beams.
- The load-carrying capacity of each specimen was governed by shear-induced anchorage failure. The length available for the development of the flexural reinforcement was limited, and cracking along the development length exacerbated the problem.

Across the state of Texas and in many other areas of the world, relatively young concrete structures have developed signs of premature concrete deterioration. In a number of cases, severe surface cracking and occasional spalling have been attributed to both alkali-silica reaction (ASR) and delayed ettringite formation (DEF). Uncertainty with regard to the structural effects of the deterioration have led engineers to err on the side of caution; costly repair or replacement schemes have frequently been implemented to eliminate long-term concerns. The Texas Department of Transportation (TxDOT) has a significant number of structures with ASR/DEF, many of which are located in its Houston District.

Most of the bridges on the US Route 59 corridor in Houston, Tex., were constructed with prestressed trapezoidal concrete box beams. After a little more than a decade in service, many of the exterior box beams are showing signs of premature concrete deterioration.

Structural tests were conducted on the dapped ends of prestressed concrete trapezoidal box beams. In addition, a structural autopsy, or an internal investigation of an epoxy-injected beam, was conducted on one of the heavily cracked beams.

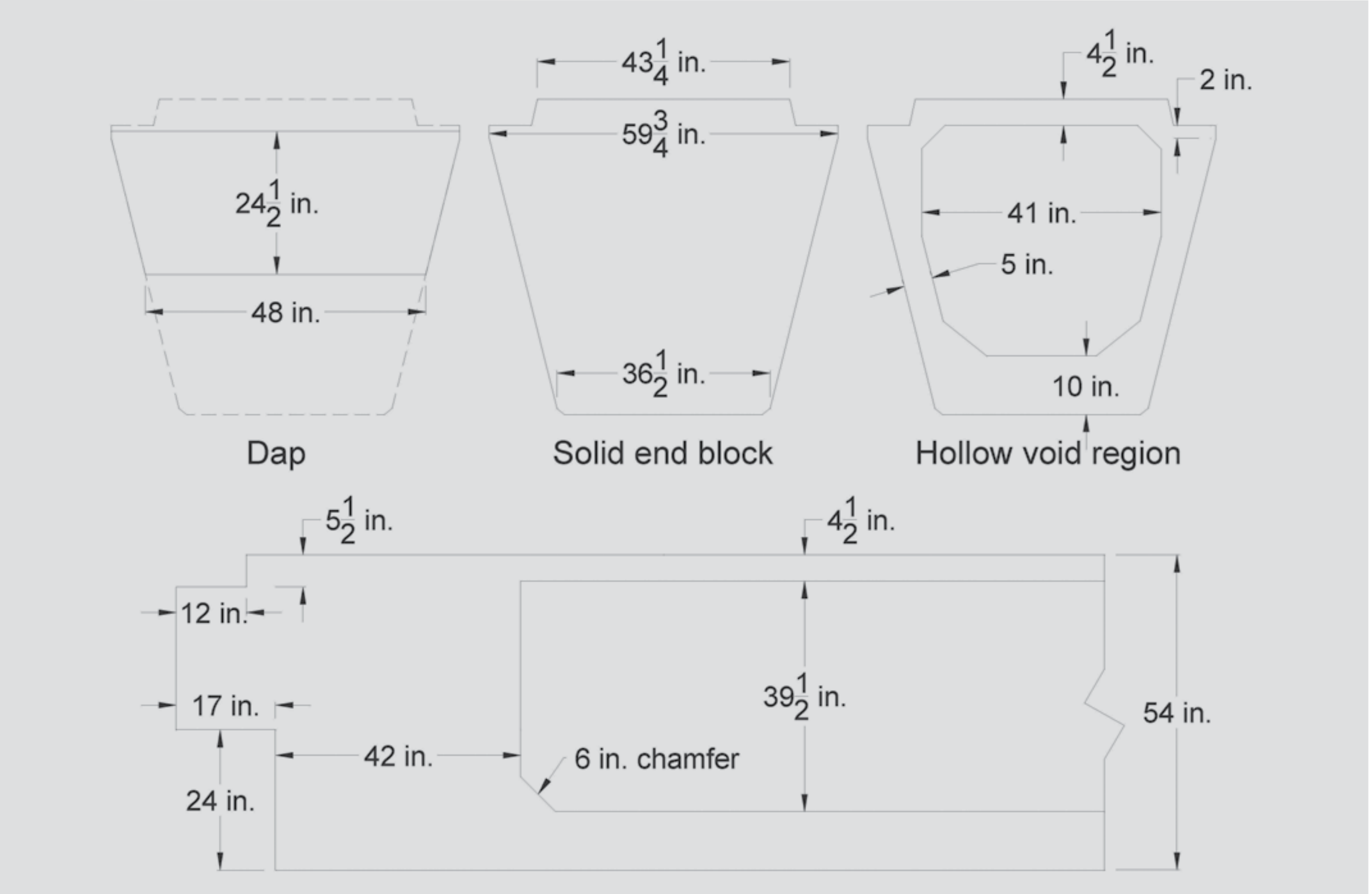

Figure 1. Dapped-end dimensions of the trapezoidal box beams. Note: 1 in. = 25.4 mm.

Research significance

ASR was first identified in the late 1930s, and significant research has been conducted on it since that time. DEF was first recognized as a potential problem in concrete exposed to high curing temperatures during the early 1980s.

Previous laboratory testing of ASR/DEF-affected reinforced and prestressed concrete beams has not revealed any significant reduction in the flexure or shear strength of simple, well-detailed elements. Although the results are generally promising, it is difficult to extrapolate the data to larger, more complex field structures. Load testing has never been performed to investigate the effects of premature concrete deterioration on the structural performance of dapped ends. The majority of research with ASR/DEF-affected members focused on smaller, rectangular sections. Test results found in the literature are unsuitable for evaluation of the unique load transfer mechanisms and reinforcement details in a dapped end.

Specimens reported in the literature were also batched with reactive mixtures chosen to reproduce in-place premature concrete deterioration. Special curing techniques were often used to accelerate the distress in the concrete. The trapezoidal box beams under investigation developed ASR/DEF damage in the normal Houston climate.

Specimen description

Five trapezoidal box beams were cast for, but not erected within, the US 59 corridor. The beams were rejected during the casting process because of rotation of the extruded polystyrene foam void and poor concrete consolidation. Over the past 15 years, the box beams have been in a precast concrete yard in Houston. Four of the beams were cast in July 1995 and showed signs of concrete deterioration (ASR and/or DEF) varying from mild to severe. The fifth beam was cast in November 1995 and showed little to no concrete deterioration; it can be considered representative of an undamaged beam. In this paper, the effects of ASR/DEF deterioration on a heavily and lightly cracked beam are compared.

Dimensions and reinforcement layout

As fabricated, the trapezoidal box beams ranged from 102 to 113 ft (31 to 34 m) in length and weighed from 65 to 71 tons (59 to 64 tonnes). To facilitate transportation to (and within) the laboratory, each of the box beams was cut into three sections to meet the hauling and laboratory crane limits.

Figure 1 illustrates the geometry of the dapped end, solid end block, and hollow void region. The segments con-

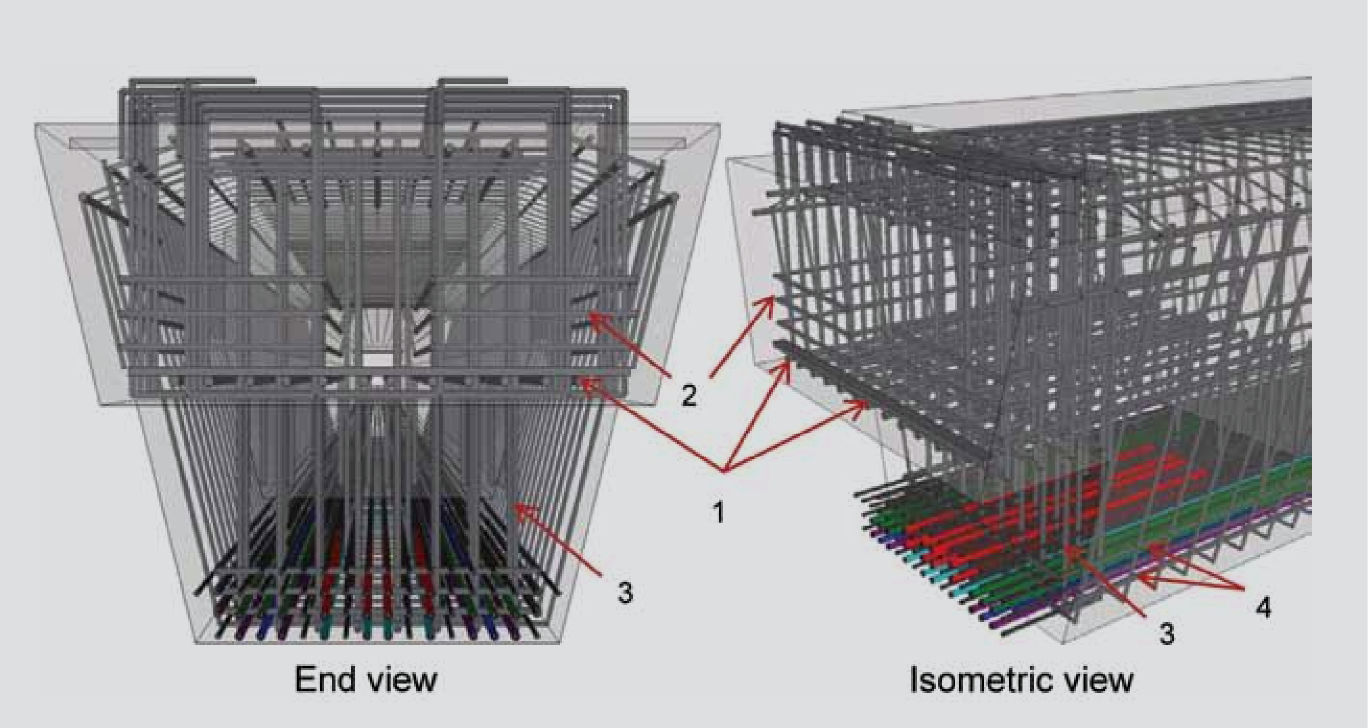

Figure 2. Dapped-end reinforcement configuration. Note: 1 = twelve no. 7 (22M) horizontal dap reinforcement were welded to a plate at the end face and anchored with a 90-degree hook 2 in. (50 mm) from the inner face of the end block; 2 = additional horizontal no. 5 (16M) hooked bars at 3 in. (76 mm) were provided up to two-thirds of the dap height to help prevent cracking at the reentrant corner; 3 = eight no. 6 (19M) vertical (hanger) reinforcement loops located near the reentrant corner extended down through the prestressing strands; 4 = three no. 6 (19M) bent bars were placed at the bottom of the beam to help provide confinement.

tained sixty-two $^1/_2$ in. (13 mm) prestressing strands with 24 strands debonded in the end region. The relatively high percentage of debonded strands (39%) coupled with the concentration of the longer debonded lengths of up to 15 ft (4.6 m) at the edges of the beam was expected to have adverse effects on the flexural and shear capacity of the member in the end block region.

The reinforcement layout (**Fig. 2**) was standard for each segment and is shown to provide a better understanding of the complexity of the members. The debonded strands are shown with various colors representing their debonded length. The original design of the beams was performed following the *PCI Design Handbook: Precast and Prestressed Concrete*,[1] and the test results indicated that the use of strut-and-tie models is appropriate for designing dapped ends of beams. Larson et al.[2] provide a detailed discussion on the strut-and-tie models used in the analysis of this project.

To prevent failure of the dap, the end block of each beam was heavily reinforced. The horizontal dap reinforcement was welded to a plate at the end face and anchored with a 90-degree hook 2 in. (50 mm) from the inner face of the end block. Additional horizontal hooked bars were provided up to two-thirds of the dap height to help prevent cracking at the reentrant corner. The large amount of vertical (hanger) reinforcement located near the reentrant corner was a feature of both strut-and-tie modeling and *PCI Design Handbook* design methods. The hanger reinforcement extended down through the prestressing strands and, along with three bent bars at the bottom of the beam, helped to provide confinement.

Premature concrete deterioration

The trapezoidal box beams with dapped ends examined in this study were affected by two premature concrete deterioration mechanisms that subjected the concrete to expansive stresses, leading to cracking on the surface (**Fig. 3**). Along with the visual assessment of ASR/DEF-related cracking, findings from a petrographic analysis were used to establish the nature of the deterioration found within the trapezoidal box beam segments. A scanning electron microscope equipped with an energy dispersive spectrometer allowed identification of microscopic features and reaction products of both ASR and DEF present in the beams.[3]

The surface cracking patterns and visible exterior damage caused by either mechanism are virtually indistinguishable from a structural engineer's perspective. A brief discussion of the chemical and physical properties of the mechanisms will be provided, but no distinction will be made between their deleterious effects in the rest of this report because both mechanisms were present in the trapezoidal box beams. The petrographic analysis did indicate, however, that ASR was the primary premature concrete deterioration mechanism responsible for the distressed concrete in the beams examined in this study.

Figure 3. Visual effects of alkali-silica reaction and delayed ettringite formation.

ASR This mechanism results from a combination of high-alkali cement and reactive siliceous aggregates in the concrete mixture. Reactive silica within the coarse and fine aggregates breaks down in the highly basic concrete pore solution and reacts with the alkalis to form a viscous gel. This gel expands as it absorbs water, generating pressure within the aggregates and hardened cement paste (**Fig. 4**). In the presence of sufficient moisture, the pressure can easily exceed the tensile strength of the aggregate particle and surrounding matrix, producing map cracking and/or surface pop-outs.[4]

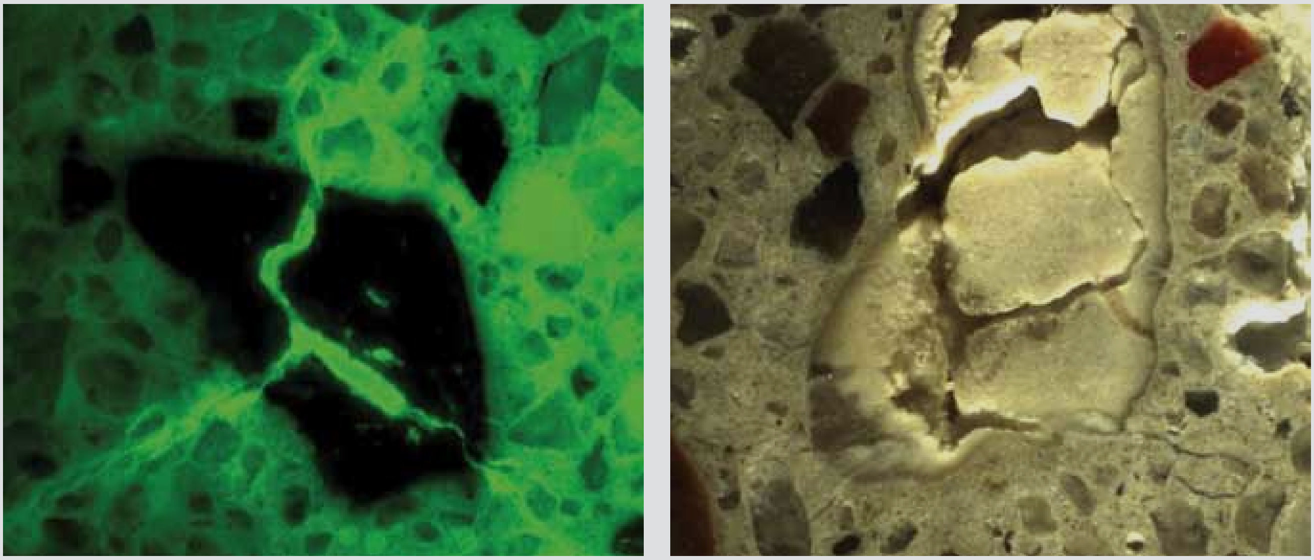

Figure 4. Fluorescent microscopic image and microscopic image illustrating alkali-silica reaction–distressed fine aggregate and associated cracking. Source: Data from Morgan (2010). Note: actual widths of images are 0.33 in. (8.41 mm) and 0.195 in. (4.95 mm).

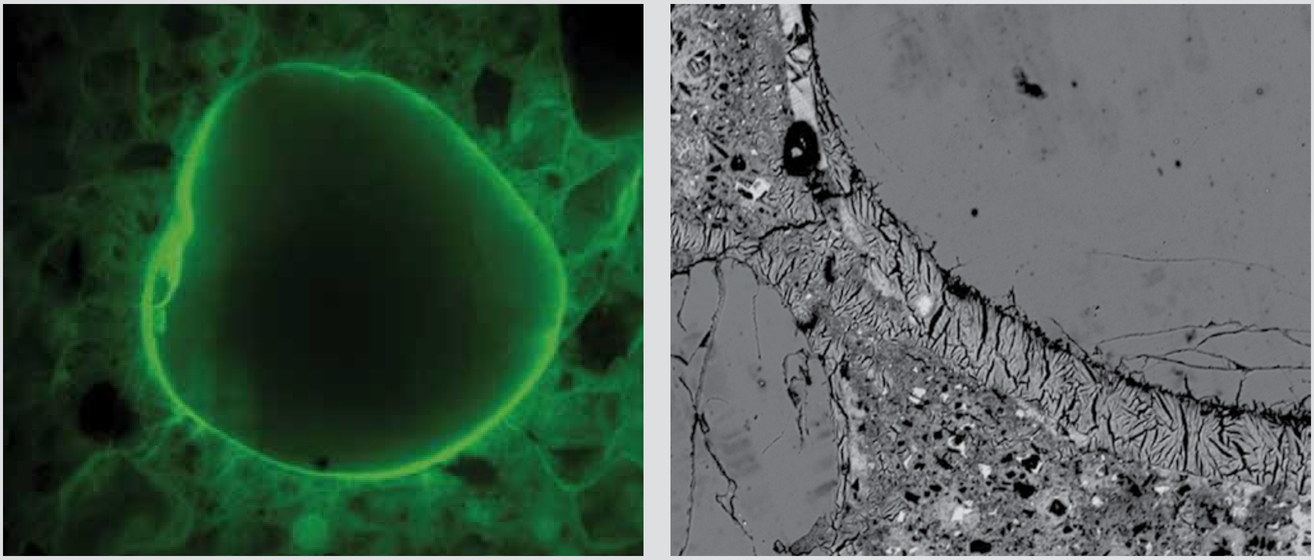

Figure 5. Fluorescent microscopic image and scanning electron microscopic image illustrating gapping around fine aggregate due to delayed ettringite formation. Source: Data from Morgan (2010). Note: actual widths of images are 0.46 in. (11.7 mm) and 0.013 in. (0.336 mm).

The expansive capability of the gel is influenced by a number of factors, including the reactive aggregate, the concentration of alkalis within the pore solution, and the availability of moisture. The reaction is highly responsive to variations in materials, mixture characteristics, and exposure conditions. This sensitivity results in significant variation of deterioration, not only between two identical members, but even within a single member.

DEF This mechanism is a form of internal sulfate attack that can occur when the concrete is subjected to temperatures in excess of 158°F (70°C) early in the curing process. Ettringite forms early during cement hydration but becomes unstable and decomposes at such high temperatures. When fresh concrete is exposed to high temperatures, the sulfates and aluminates form monosulfoaluminate, which becomes trapped within the early cement hydration products. Over time with the availability of sufficient moisture, the sulfates and aluminates diffuse out of the hydration products to react and form ettringite. The re-formation of ettringite produces expansive forces and microcracking of the hardened cement paste.[4]

The growth of ettringite leads to bulk expansion of the cement paste and the development of cracks and gaps around the aggregates (**Fig. 5**). The ettringite then proceeds to fill the recently formed cracks and create rims surrounding the aggregates, furthering the overall expansion and crack development. The formation of large amounts of ettringite within the hardened cement paste can cause expansions of magnitudes well in excess of those due to ASR, and wide variations in cement composition (sulfate content), mixture characteristics (porosity), and exposure conditions will lead to a similar variation of deterioration.[4]

Segment conditions

The cracked surfaces of the beams were assumed to represent the future condition of the bridge beams currently in service in the next 10 to 30 years, depending on exposure conditions and coatings that may be applied to the structures. This assumption was based on the fact that the beams in the precaster's yard were left uncovered and thus directly exposed to high temperatures and moisture. The deleterious chemical mechanisms ASR and DEF occur at a slower rate in the actual bridge beams because of the protection from the weather provided by the reinforced concrete bridge deck. Not only were the beams in the yard exposed to the elements, they were also found to have moisture trapped in the extruded polystyrene foam core.

Figure 6 shows crack patterns in the two specimens compared in this paper. Varying levels of ASR/DEF-related damage were chosen to determine the effects of cracking on the strength of the dapped ends. The level of distress in each beam, determined by comparing the extent and the width of the cracks, was designated as light and heavy cracking. The maximum widths of the major cracks are shown in Fig. 6 to validate the damage assessment. These crack widths ranged from 0.016 in. (0.41 mm) in the prestress transfer region of the lightly distressed specimen to 0.25 in. (6.4 mm) in the beam that experienced heavy cracking.

Box beam autopsy

A forensic investigation of one of the trapezoidal box beams provided valuable information on the physical effects of ASR/DEF. The box beams were fabricated using standard industry practices and were allowed to deteriorate under storage conditions in the hot, moist Houston envi-

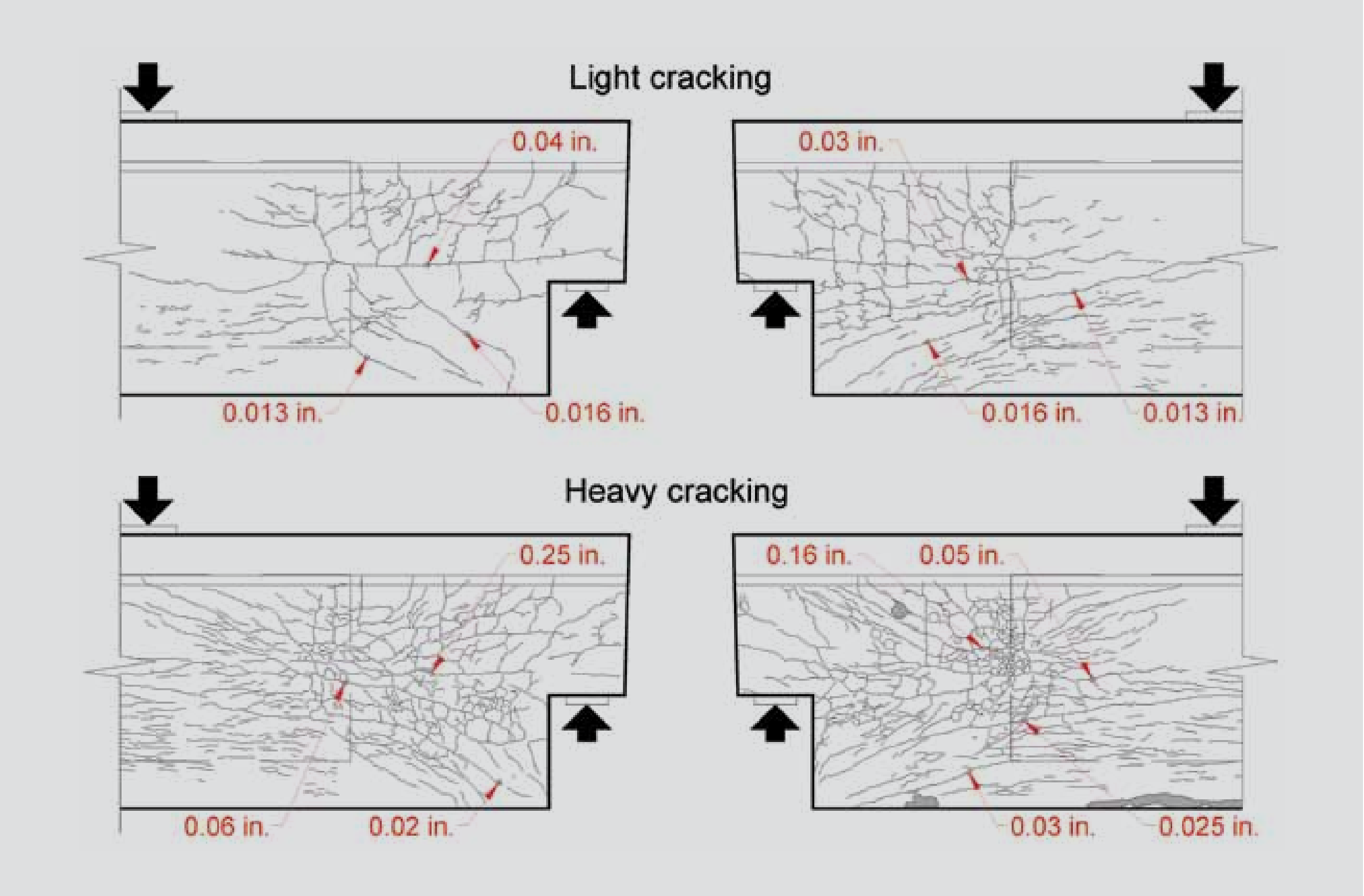

Figure 6. Alkali-silica reaction and delayed ettringite formation–related damage to dapped ends. Note: 1 in. = 25.4 mm.

ronment. The average high temperature ranged from 95°F (35°C) in the summer to 60°F (16°C) in the winter. The low temperature ranged from 75°F to 45°F (24°C to 7°C). The average relative humidity ranged from 90% in the morning to 55% in the afternoon. Therefore, the beam provided a unique opportunity to directly inspect the internal damage resulting from nearly 15 years of continuous deterioration. A segment with a standard end block exhibiting heavy deterioration was selected for the structural autopsy. The deterioration was quite severe compared with most of the segments. There were wide cracks, small areas of spalling, and efflorescence at some of the cracks.

To clearly identify penetration of the surface cracks into the cross section, the standard end block was injected with epoxy prior to placing the cuts. Once the outer epoxy seal had adequately cured, a black-colored epoxy was injected through carefully placed ports. Sufficient penetration into the crack network was established when a threshold injection pressure was met. The segment was then moved into the lab where cuts perpendicular to the longitudinal axis of the box beam were made with a concrete wire saw.

Contrary to the outward appearance, the surface cracks did not penetrate deeply into the structural core of the member. In fact, all of the cracks terminated within 10 in. (250 mm) of the beam's surface. The most notable cracks ran parallel to the left side of the cut faces (**Fig. 7**). This interior cracking corresponded to the more heavily cracked face of the box beam segment. While the wide parallel cracks occasionally intersected surface cracking, they did not appear to be a direct extension of discrete surface cracks. It is more likely that the internal cracking was the result of the presence of highly localized restraint in the vicinity of the transverse reinforcement and the lack of compatibility between the confined structural core and free concrete surface. While the transverse reinforcement provided confinement to the structural core, the cover concrete was free to expand away from the core.

In general, the internal cracks diminished in width and length as the longitudinal distance from the free end of the box beam segment increased. Cracking parallel to the beam's surface was up to 0.40 in. (10 mm) wide within the first section (14 in. [360 mm] from the free end), while the cracks found within the section 55 in. (1.4 m) from the free end were up to 0.08 in. (2 mm) wide. The observed trend cannot be attributed to variation of the reinforcement details because the transverse reinforcement was consistently spaced at 5 in. (130 mm) over the length of the region under consideration. It is likely that the gradient of temperature (and corresponding damage potential) present at the transition from the solid to hollow sections contributed to the diminishing severity of the internal cracking.

Figure 7. Typical alkali-silica reaction and delayed ettringite formation–related defects in end block cross sections.

The cut face intersected the transverse reinforcement in two of the cross sections (Fig. 7). In both cases it appeared as though the cracking led to debonding of the back side of the transverse reinforcement by the black epoxy in the center picture. The debonded length was relatively short in both cases but could have affected the splice in the vertical bars at that location. Despite the observation, the beams did not exhibit stirrup anchorage failure during testing, so it can be assumed that a similar defect in other box beams might not affect overall structural performance.

With regard to the structural effects of the interior damage, fine cracking between the prestressing strands was identified as the most significant aspect of the deterioration. Because of the rapid transfer of prestressing force from the strands to the concrete, tensile (or splitting) stresses were generated in the transverse plane of the beam when the strands were released. Although the magnitude of the tensile stress at transfer should not have caused splitting cracks, it is likely that it was augmented by tensile stresses related to ASR/DEF actions and resulted in cracking in the horizontal plane of the strands. Close examination of the exposed strand ends revealed fine cracking between a number of strands in each layer (Fig. 7). The interstrand cracking generated by ASR/DEF expansion was suspected to be responsible for the observed loss of anchorage and overall dapped-end capacity that is discussed later on.

Testing procedure

Figure 8 shows the setup used for the dapped-end testing program. The beams were simply supported on each end with a single point load applied at 99 in. (2500 mm) from the support, resulting in a shear span–to–depth ratio of 1.85. The testing frame was composed of a transverse spreader beam attached to two 2000 kip (8900 kN) hydraulic rams, each with a 12 in. (300 mm) spherical head to account for slight misalignments. Two 4 in. thick (100 mm) steel plates with dimensions of 26 × 24 in. (660 × 610 mm) evenly distributed the load to both box beam webs.

The ends of each box beam were supported by a total of three bearing plates, one at the dapped end and one under each web at the other end of the beam. The dapped-end bearing plate measured 32 × 9 in. (810 × 230 mm) and was placed with its long side perpendicular to the longitudinal axis of the beam. The other bearing plates measured 16 × 9 in. (400 × 230 mm) and were placed under each web parallel to the longitudinal axis of the beam.

Once the segment was positioned in the test setup, a 4 in. (100 mm) topping slab was cast as would have been the case in the field. The ASR/DEF-related surface cracking was marked and thoroughly documented through photographs and crack width measurements. Load increments of 100 kip (445 kN) (total load as measured by the load cells) were applied until the first new crack was observed. The increments were then decreased to 50 and 25 kip (220 and 110 kN) to carefully observe the crack progression and accurately record critical loads. Once the shear crack widths exceeded 0.06 in. (1.5 mm), load was steadily increased to failure.

Results and discussion

One of the major concerns with the prestressed concrete trapezoidal beams was the capacity of the extended portion (dap) because it had a smaller section to carry a large shear force (end reaction). Therefore, it was thought that the dap would govern the strength of the beam. Through load testing it was found that the strength of the smaller section did not control, and it was concluded that a dap failure was not

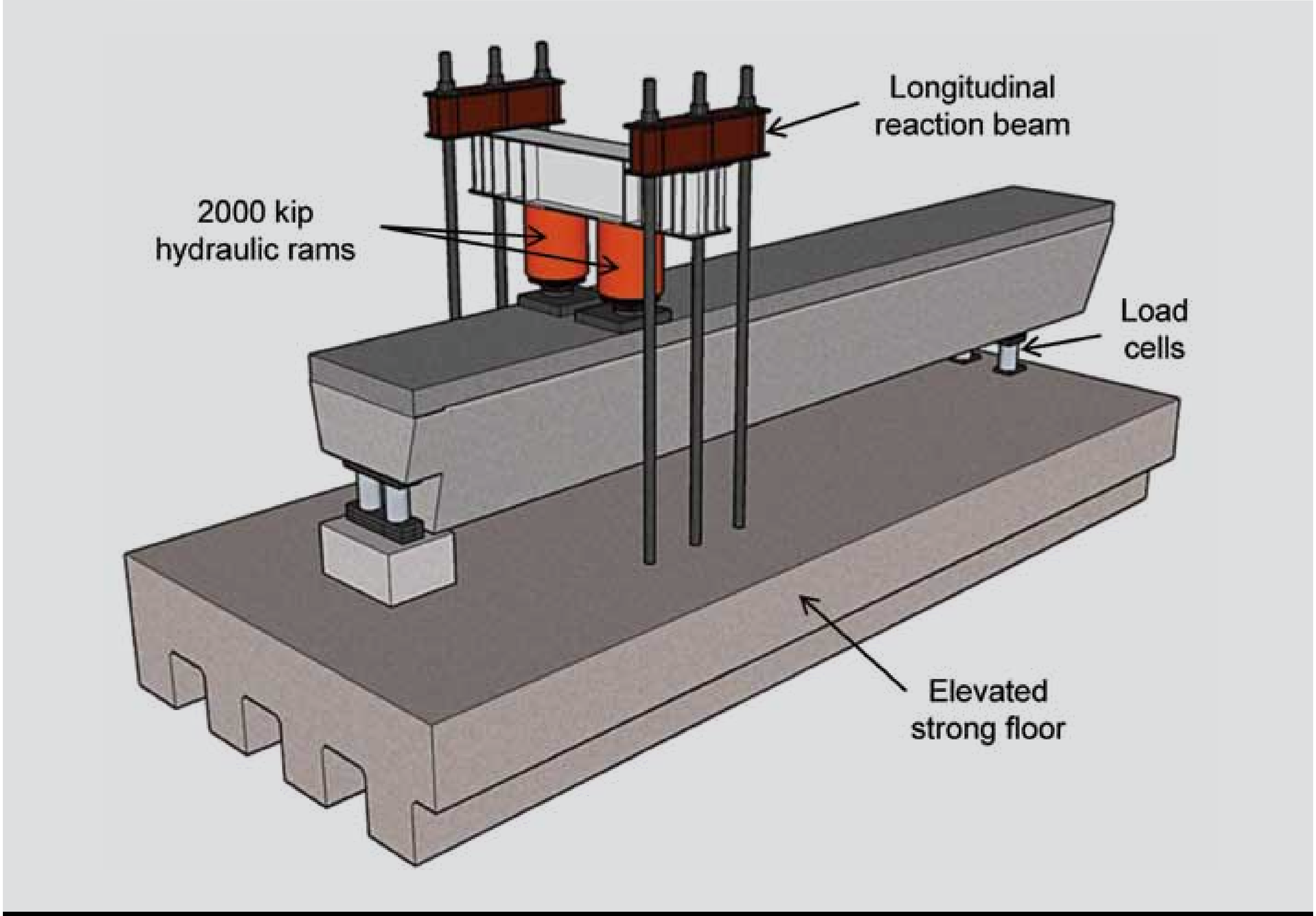

Figure 8. Test setup. Note: 1 kip = 4.448 kN.

likely for the specimens in this study because of the presence of heavy reinforcement in the extended portion of the beam. Based on test observations, failure of the US 59 dapped-end trapezoidal box beams was determined to be governed by anchorage of the mild and prestressed flexural reinforcement.

Loading of the lightly and heavily cracked beams at a shear span–to–depth ratio of 1.85 resulted in shear-induced anchorage failures. In general, diagonal cracks (preexisting and load-induced) extended into and opened within the transfer length of the prestressing strands (**Fig. 9**). The cracks have been highlighted for the benefit of photographs and identifying the formation of new shear cracks during testing. Subsequent extension of the diagonal cracks across the bottom of the box beam was accompanied by minor spalling in several of the beams. A loud pop signified significant slip of the prestressing strands and was followed by a drastic loss of load-carrying capacity. In most cases, the shear-induced anchorage failure was accompanied with extension of preexisting ASR/DEF cracks.

The heavily cracked segment produced the most sudden, pronounced failure of the damaged dapped ends (Fig. 9). Cracking at failure followed preexisting ASR/DEF cracks between the load and bottom corner of the beam, forming a large, continuous crack that widened extensively and extended into the deck under the hydraulic rams and under the beam in the strand anchorage area. The maximum applied shear resisted was 659 kip (2930 kN). The lightly damaged segment failed at 703 kip (3130 kN).

The linear-elastic response extended to 70% or 80% of the maximum applied shear (**Fig. 10**). After that level, the stiffness reduced as a result of preexisting crack growth and new crack development. The most significant aspect of the load-deflection plot of the distressed dapped ends is the greater stiffness of the more damaged beam.

It is likely that restraint of the ASR/DEF-related expansions by the dapped-end reinforcement led to the development of compression within the structural core concrete, eliciting a stiffer material response. Concrete cores were removed and tested to estimate the strength in each beam, and it was found that the heavily cracked beam had a lower compressive strength than the lightly cracked beam. ASR/DEF-induced compression has been shown to be capable of offsetting (and superseding) the loss of mechanical strength or stiffness due to the microstructural cracking in previous research as well.[5]

The presence of ASR/DEF-induced compression also provided a logical explanation for the observed delay of first

Cracking in the transfer length

Bottom-side anchorage cracks

Postfailure widening and extension of alkali-silica reaction and delayed ettringite–formation cracks

Figure 9. Typical features of alkali-silica reaction and delayed ettringite formation–damaged segment failure.

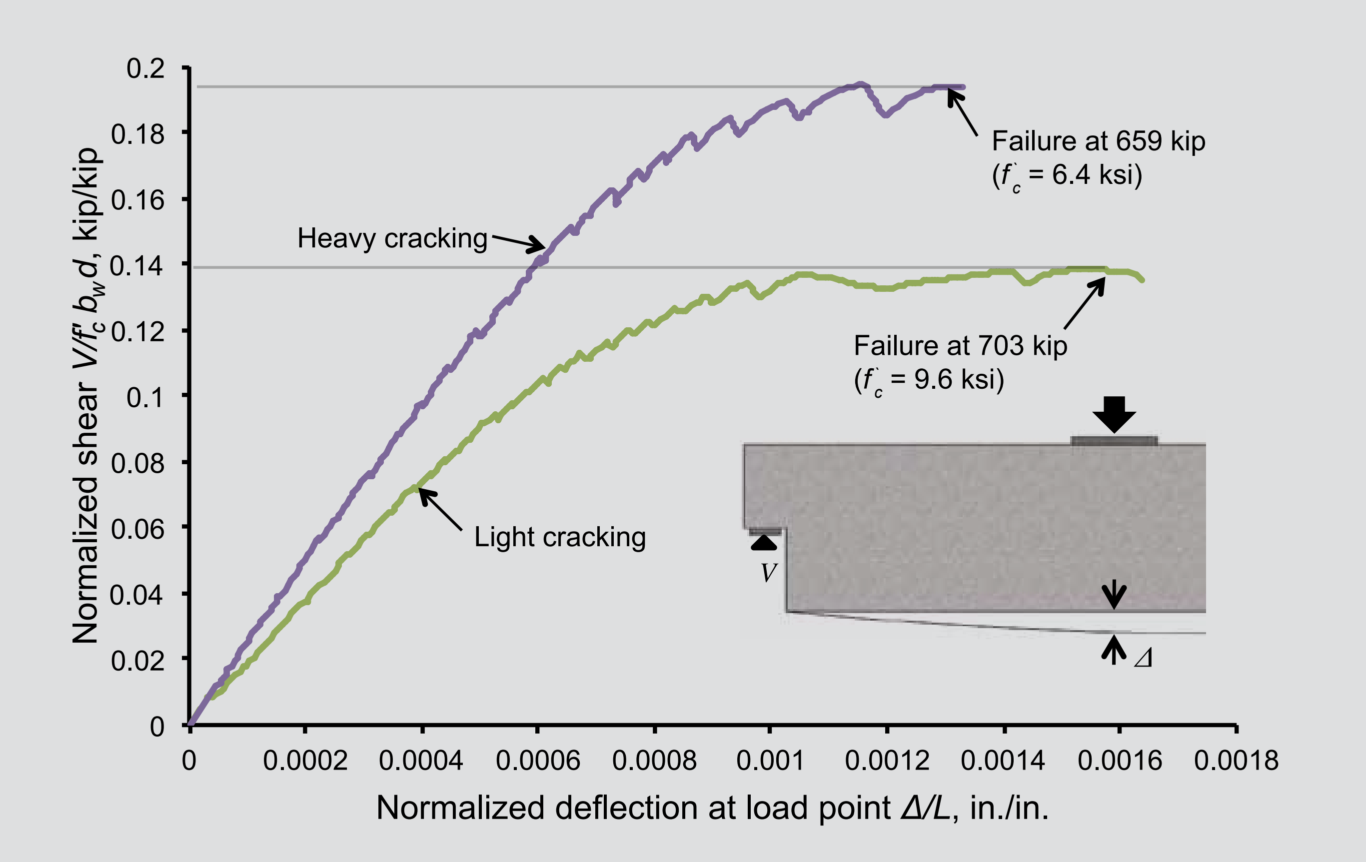

Figure 10. Load-deflection responses (normalized shear stress). Note: b_w = web width; d = beam depth; f'_c = concrete strength calculated from core strength; L = beam length measured from center of supports; V = total shear measured by load cells plus the appropriate calculated self-weight of the specimen; Δ = deflection measured at the load point. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

cracking in the damaged segments. **Figure 11** shows the development of load-induced cracking in the lightly and heavily damaged segments. The illustrations cover the full range of behavior noted during the tests. The response of the segment with light cracking was similar to an undamaged segment. First cracking occurred at a relatively high percentage of the capacity (67%), but an extensive crack network was nonetheless present before failure. The behavior of the heavily cracked beam (subject to 0.25 in. wide [6.4 mm] cracks) was drastically different. Well-defined shear cracking did not develop at any point in the dapped-end test, including failure. It is possible that the cracking was distributed in the strut due to the ASR/DEF distress, but no significant new crack formation was observed.

The only notable crack on either face of the heavily damaged segment (Fig. 9) corresponded to a preexisting ASR/DEF crack. Overall, the results suggest that either greater ASR/DEF-related damage led to the development of higher internal compressive stresses that further delayed the formation of load-induced cracking or that there was no need for more cracks to form because extensive cracking already existed. Briefly stated, the serviceability characteristics of the damaged dapped ends generally supported the assertion that the expansion-induced compressive stresses within an ASR/DEF-affected member can effectively offset (and potentially negate) the structural effects of reduced strength and stiffness in plain concrete.[5]

Historically, the considerable loss of elastic modulus and tensile strength in plain concrete has led to concerns regarding the strength of distressed concrete structures. A review of simple beam tests conducted over the past three decades revealed that the ASR/DEF deterioration did not lead to a measureable loss of strength or stiffness in flexure or shear.[5]

The anchorage of mild reinforcement and prestressing strand, however, does not benefit from ASR/DEF-induced compression. Many of the strands in the trapezoidal box beams are located away from the transverse ties. The concrete expansion mobilizes the ties but does not help to reduce cracking between the strands or the loss of cover through spalling. At a certain point, it appears that the preexisting cracking in the development and transfer region can negate any benefit gained from the expansion of the concrete. Application of load increases the width and extent of the cracks, further reducing bond between the concrete and steel and resulting in anchorage failure.

The shear strength of the heavily damaged beams was relatively unaffected despite the large cracks in the end block.

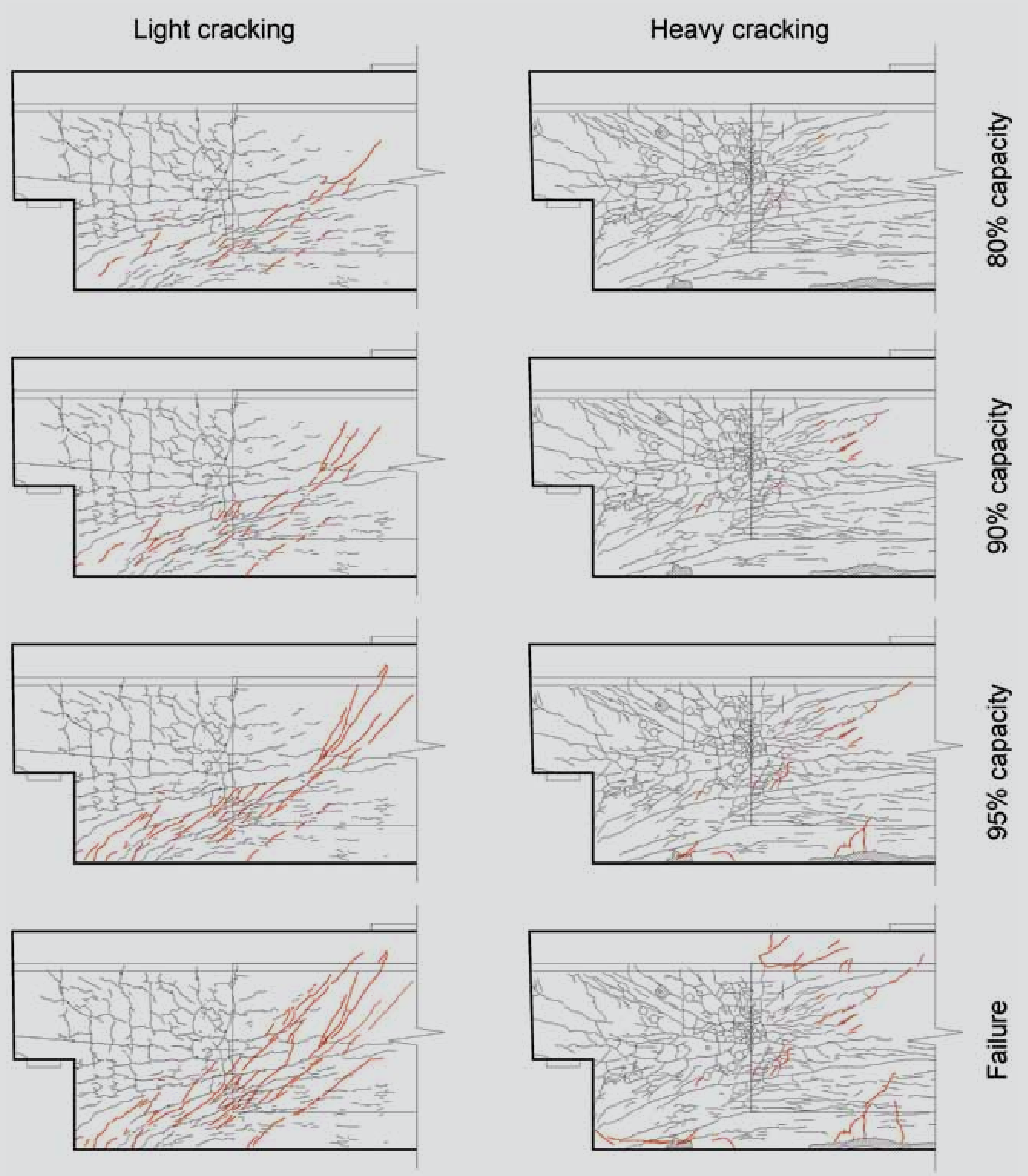

Figure 11. Diagonal cracking on the lightly and heavily damaged specimens.

These beams failed due to shear-induced anchorage failure. It was not the strength of the concrete but the anchorage of the prestressing strands that controlled the failure of both beams. Based on the preliminary analysis of the results, it appeared that premature concrete deterioration weakened the anchorage of the flexural reinforcement in the dapped-end region. The lightly damaged segment supported a higher shear (703 kip [3130 kN]) than the heavily cracked beam that reached only 659 kip (2930 kN).

The failure mode of the beams highlights the critical nature of the confinement provided by well-anchored reinforcement in ASR/DEF-affected structures. Loss of the confinement (perhaps through fracture of the stirrups) would have severe implications for the integrity of damaged structures. The results suggest a loss in strength through anchorage failure with increasing deterioration. The difference in the capacity of the two beams was relatively small, about 6%, but the lightly cracked beam had most likely already suffered a reduction in anchorage capacity due to ASR/DEF distress. Furthermore, any additional ASR/DEF distress beyond that of the heavily cracked specimen in this study could result in values below the calculated dapped-end design capacity. Larson et al.[2] discuss additional beam tests and calculated capacities.

Conclusion

The observations and data gathered over the course of the study provide a clear picture of the relationship between the severity of the ASR/DEF deterioration and the structural performance of the dapped-end beams.

The results of the tests are in agreement with the findings of previous projects. The flexural and shear capacity of the trapezoidal box beams were not greatly affected by the ASR/DEF deterioration. However, the structural capacity of the dapped ends was governed by shear-induced anchorage failure. The initial attempt to fail the dap (partial-depth portion) in shear resulted in anchorage failure of the prestressing strands. Detailed review of the beam design revealed the critical nature of the flexural reinforcement anchorage. Due to the relatively short length available for prestressing strand development, this detail will likely control failure of the beam end irrespective of the shear span–to–depth ratio.

Two particular characteristics of the damage suggested that ongoing ASR/DEF deterioration was responsible for the loss of anchorage capacity. First, relatively large surface cracks (up to 0.03 in. [8 mm] wide) were identified within the prestress transfer region of all of the deteriorated segments. Second, structural autopsy of one box beam segment revealed fine cracks between the prestressing strands.

The findings will result in the development of better repair and replacement recommendations for trapezoidal box beam bridges as well as other structures in which ASR/DEF-related cracking is evident.

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The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of TxDOT.

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Notation

b_w = total web width

d = beam depth

f_c' = concrete strength calculated from core strength

L = beam length measured from center of supports

V = total shear measured by load cells plus the appropriate calculated self-weight of the specimen

Δ = deflection measured at the load point

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Abstract

Concern about the effects of alkali-silica reaction (ASR) and delayed ettringite formation (DEF) on dapped prestressed concrete trapezoidal box beams in Houston, Tex., prompted an investigation of their shear strength. Beams originally intended for a highway bridge project were stored in a local precast concrete yard and were exposed to the environment for 15 years. During that period, ASR/DEF deterioration of varying severity occurred.

Load tests were conducted on five specimens (two of which are discussed in this paper) with varying levels of distress to study the performance of the dapped end and to assess the effect of the ASR/DEF deterioration on the in-service beams. The strength of each specimen was governed by shear-induced anchorage failure confirmed both by the formation of cracks in the development region and slip of the prestressing strands. The length available for the development of the flexural reinforcement was limited, and ASR/ DEF cracking along the development length exacerbated the problem.

Keywords

Alkali-silica reaction, dapped end, delayed ettringite formation, research, shear, strand anchorage.

Review policy

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